

CALIFORNIA DEPARTMENT OF TRANSPORTATION  
EARTHQUAKE ENGINEERING  
SEISMIC DESIGN COMMENTARY

By Ray Zelinski

July 1995

Several seismic related design or construction issues have surfaced in the past several months. Following is a commentary on those issues:

1. Seat extender openings. Cored and formed holes for pipe seat extenders should be enlarged to 10 inches when elastomeric pads are used as hinge seat bearings. The Office of Structures Maintenance has requested the increase to provide added vertical clearance for jacking in the event bearings need replacement.

2. Hinge Restrainers. Hinge restrainers should be compatible with hinge displacement capacity. Details showing type C1 restrainers, or equally restrictive restrainers, in combination with seat extenders have been observed in recent months. Those type restrainers have limited strain capacity and will probably rupture before the extenders can be mobilized. This non-complementary series type condition is less efficient than a parallel system using more ductile restrainers with seat extenders. Therefore, the series type condition should be avoided.

A related situation is where existing restrainers are grouted in a pipe. This type restrainer performed poorly in the Northridge Earthquake because the wrapped cables did not slip within the grout and permit expected strains. Brittle tensile rupture resulted. All grouted restrainer systems should be replaced. If the existing type is not known, a field determination should be made, or a letter to the R.E. Pending File requesting construction confirmation or identification should be issued.

When existing restrainers will be retained, they must be inspected for proper gapping and positive anchoring methods (i.e.: Loctite on threads). Also, new restrainers, added to existing ones as part of a retrofit scheme, should be strain compatible to avoid premature rupture in any cable before full yield can be achieved in all restrainers.

There have been some instances when existing restrainers are abandoned, but do not interfere with new construction details. The designer may choose to leave the existing restrainers in place, with or without anchorage intact. The designer must assure that there will be no adverse structural affect on the hinge components or thermal movements as a result of this choice.

3. Dowels inserted in Bottom Surfaces. There has been some recent use of dowels inserted in the underside of bent caps for infill walls. This situation, and any situation requiring doweling into the bottom surface of a member, should be avoided if at all possible. It is very difficult to perform the drilling and anchoring. Furthermore, it is extremely difficult to avoid reinforcement during the drilling operation and thus obtain a good dowel pattern in heavily reinforced members such as a bent cap. Soffit dowel details and installation procedures must be carefully selected and prescribed if their use cannot be avoided. Also, if infill walls are detailed, they are usually for transverse load control and it is usually not necessary to construct them up to the bent cap soffit. A 6-inch gap helps constructability. There may be times when the designer needs to provide bearing, in which case the gap cannot be specified.

4. Dowels into Piles through Pile Caps. There was at least one case of inserting dowels into piles by drilling through the pile cap. This operation has a low probability for success. Avoid use of this detail.

5. Column Reinforcement splicing. Current Policy specifies no splices in main column reinforcement in plastic hinge zones. Recent qualification and quality control tests at Translab for both mechanical and welded splices corroborate the wisdom of this policy. Contact the Seismic Technology Section or Reinforced Concrete Technical Committee Chairman if you need to consider straying from this policy for a specific circumstance. Always consider an alternative to splicing in the plastic hinge zone as a first solution to a problem.

Splicing spirals in plastic hinge zones usually cannot be avoided. The standard Caltrans welded spiral splice detail should be used for plastic hinge zones. The Caltrans standard welded hoops are permissible in plastic hinge zones.

The standard Caltrans spiral splice has not been substantiated for spiral sizes greater than #6 bars. Transverse column steel, in sizes greater than #6, should be welded hoops.

Project detail sheets must clearly state zones for splicing or not splicing, types of rebar (i.e.: longitudinal or transverse) to be spliced or splices prohibited, and types of splices allowed or prohibited.

All of the splicing policies are being reviewed and, therefore, are subject to change. Be aware that some of the preceding instructions may soon be revised.

6. Seismic Hooks. When seismic hooks (i.e.: 135o hooks in potential plastic hinge zones or potentially cracked torsional sections) are required for stirrups or ties, the hook tails must be dimensioned on the plans. Standard 135o hook tails, which are shorter than the seismic version, will be fabricated unless detailed otherwise on the plans.

7. Assumed Pile Tension Capacity. Generally 50% of the ultimate compressive capacity of friction piles is allowed for uplift, and no tensile capacity is allowed for end bearing piles. However, there are other situations for which the designer must determine applicable allowables.

Two non-standard situation examples are:

- A. Piles which attain compressive capacity partially from both friction and end-bearing. In this case, seismic uplift capacity must be limited to 50% of the friction portion.
- B. Piles in soft cohesive soils which attain compressive capacity from end bearing. Tests have shown that the soft soils provide significant uplift resistance which can be mobilized for transient loadings such as seismic.

The Engineering Geology Section should be consulted for allowable pile seismic tension capacity in all cases. Also, the designer is reminded of the responsibility to assure that the pile connection to the footing and its structural tensile capacity is consistent with the estimated geotechnical tensile capacity.

8. Blocking Abutment Gaps. When specifying blocking in the gap between the abutment diaphragm and backwall, the designer should specify the required seasonal thermal clearance, not the blocking thickness dimension. The gap dimension often varies, therefore it is nearly

impossible to specify a blocking thickness which guarantees appropriate thermal and seismic gapping. An approximate gap dimension should be detailed on the plans for bidding purposes.

The designer is advised to consider concrete or steel tubing filler in lieu of timber in the gap. There has been evidence of campfires under bridges in the vicinity of abutments. Timber fillers are at risk if a fire is present.

9. Existing Pile Lateral Capacities. In the March 1993 issue of "What's Shakin'", pile lateral loads and stiffness were supplied. Pile sizes and types were not clearly identified. Also, the steel piles tested were pipe piles and recommendations for "H" piles were not provided. The table from "What's Shakin'" has been revised and attached to these instructions for your use.

10. Lap-spliced Spiral Strain Allowable. A conservative value for allowable strain in lap-spliced spirals should be assumed when calculating column plastic displacement capacity. Some pre-1980 spirals could have been lap-spliced without hooks. Lap-spliced spirals without hooks should not be allowed the same strain capacity associated with fully confining continuous spiral. The lack of reliability was demonstrated recently at the Santa Monica Viaduct retrofit project. When column surfacing was removed to construct link beams, the spiral unraveled without prodding. This situation is not unlike expected spalling during development of plastic hinging in an earthquake.

In a related matter, designers should not specify surface removal of a column as a preparation for a retrofit installation if lap-spliced spiral is suspected.

11. Allowing Yield in the Superstructure. Retrofit strategy often allows yielding in the superstructure, transversely, longitudinally, or both. The designer has an obligation to assure that the analytical model is consistent with that strategy and that a reliable load path is provided for all imposed load demands. The yielded superstructure can be assumed fixed if the ductility demand is 1.5 or less. It should be assumed pinned if the ductility demand exceeds 1.5. The analytical model must be revised as required, similar to pinning a column base which is allowed to fuse at a lap-spliced connection. Furthermore, vertical gravity and seismic loads need to be resisted through the cracked section. As a result of recorded ground motions in the Northridge Earthquake, 1.5G is the recommended minimum vertical design load. When fixity exists ( $D.D. < 1.5$ ), a combination of concrete and steel reinforcement can be used to calculate capacity. When a pin condition exists, only the reinforcement through the assumed crack zone should be used to calculate capacity. Bridges located near a fault are expected to receive a greater than normal vertical shock.

12. Superstructure Capacity Analysis. Designers may choose to analyze the superstructure and design the necessary reinforcement for seismic resistance in new bridges in lieu of using the empirically prescribed areas and lengths. This choice is most logical when reinforcement lengths resulting from the prescribed empirical method exceed the standard 60 feet. In the longitudinal analysis, the moment is assumed to be spread across an increasing width of superstructure proportional to the distance from the bent cap. The increase in width is equal to twice the distance from the cap (i.e.: the section widens assuming a 45-degree spread each side of the critical section at the bent). The designer is reminded that the column plastic hinging demand must be satisfied on only one side of the column when an expansion hinge is located in the superstructure on the opposite side.

There has been some confusion about summation of capacities and demands in the superstructure, especially in prestressed bridges. The designer must pay careful attention to algebraic signs. For prestressed bridges, the secondary moment is treated as a demand on the section, as are gravity and column plastic hinging loads. The prestressing steel and conventional reinforcement combine to provide resisting capacity.

13. Shear Capacity of Spiral. Periodically there seems to be some confusion regarding the area of steel in the shear strength expression for spiral or circular hoops:

$$\frac{\lambda A_s f_y D}{2 S}$$

As is the area of the spiral bar size. It is not the area of two legs of the spiral. The definition of the terms in the Bridge Design

Specifications are quite explicit.

14. Railroad Underpasses. See the Seismic Technology Section if you are designing a retrofit for an underpass. The University of Nevada recently completed push tests on an abandoned underpass in Los Angeles. Although the final report will not be available for several months, capacities associated with fused sliding bearings, continuous rails, ballast, etc. are available. Those factors could reduce or eliminate retrofit needs.

15. Bent Cap Reinforcement. In the design memo authored by Ray Zelinski, dated January 1995 and issued by Mellon and Post on February 14, 1995, there are a few items which require clarification. They are: 1) There is an error in the cap detail for skews > 20°. The bottom slab reinforcement is shown incorrectly. That reinforcement should be detailed in accordance with Bridge Design Details Manual pages 7-43 and 7-43.1. 2) The vertical steel requirement of 20% column steel shall be distributed uniformly within the prescribed zone, but not within the column core. 3) The 10% side face reinforcement instructions need correcting. The area of side face reinforcement should be 10% of the cap tension reinforcement at the column. It should be distributed uniformly along both cap side faces. Other provisions for side face reinforcement in Bridge Design Specifications Article 8.17.2 are also applicable.

16. Temporary Shoring Carrying Live Load. Lateral support for temporary shoring when columns or bearings are being removed and replaced, and when traffic is flowing on the bridge, must be designed to satisfy a minimum level of seismic and other group loadings. Refer to the attached guidelines for recommendations.